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Name Of Dam: SMITH RIVER DAM

Location: HENRY COUNTY, VIRGINIA

Inventory Number: VA. NO. 08913

LEVEL II

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

SCHNABEL ENGINEERING ASSOCIATES, P.C.
J. K. TIMMONS AND ASSOCIATES, INC.

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

(1)

NAME OF DAM: SMITH RIVER DAM
LOCATION: HENRY COUNTY, VIRGINIA
INVENTORY NUMBER: VA. NO. 08913

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

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(10) James A. Walsh

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PREPARED FOR
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(15) DACW65-79-D-ØØØ4

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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PHASE I - INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Nameof Dam: Smith River Dam, Va. No. 8913

State: Virginia

County: Henry

Coordinates: Lat 36°-39.9' Long 79°-53'

USGS Quad Sheet: Martinsville

Stream: Smith River

Date of Inspection: May 1, 1979

BRIEF ASSESSMENT

Smith River Dam is a stone masonry, gravity structure approximately 538 ft long consisting of a 322 ft principal spillway, a 144.5 ft auxiliary spillway, and a 71.5 ft non-overflow section. There are 6 ft high steel crest gates along the principal spillway section and 3 ft high flash boards along the auxiliary spillway. There are two 15 ft wide intake structures which supply the turbine generator adjacent to the dam.

The dam is located on the Smith River about one-half mile south of the Martinsville, Virginia City Limits and is owned and operated by the City of Martinsville as a hydro-electric facility. Construction was completed in 1904± with major revisions in 1931. The dam is located downstream of Philpott Reservoir and flow has been regulated since 1950 for over half of the watershed. The maximum known flood was in 1938 when it is estimated there was 10 ft of water over the dam crest.

The dam is of "intermediate" size and is rated

a "high" hazard structure. The "high" risk category requires that the spillway pass the PMF. Analysis indicates that the dam will be overtopped during ½ PMF. The principal and auxiliary spillways are thus inadequate since they will not pass the spillway design flood. The principal and auxiliary spillways will pass 18% of the PMF before exceeding elevation 703 msl. This dam was designed to be overtopped during periods of high flow. A check of the stability in accordance with the Corps of Engineers' guidelines, assuming the dam is founded on the surface of the rock, indicates the structure does not meet the overturning and sliding requirements of Reference 1, Appendix V, for normal pool elevation 696. An accurate check on stability could not be made since design data and calculations were not available concerning the dam embedment. However, based on visual inspection and the service record of the dam since 1932, additional studies are not recommended.

In general, the overall condition of the dam appears to be good. The visual inspection revealed the need for the following maintenance and monitoring measures. The stone masonry in the left auxiliary spillway of the dam has deteriorated. All loose and weakened mortar should be removed and the surface repaired with new mortar. The crest gates show leakage in several locations and the concrete sill should be repaired. Seepage in the plugged openings of the left auxiliary spillway should be monitored quarterly

to detect any increase in flow rates. A staff gage should be installed to monitor high water levels.

Submitted By:

Original signed by
JAMES A. WALSH

James A. Walsh, P.E.
Chief, Design Branch

Approved:

Original signed by:
Douglas L. Haller

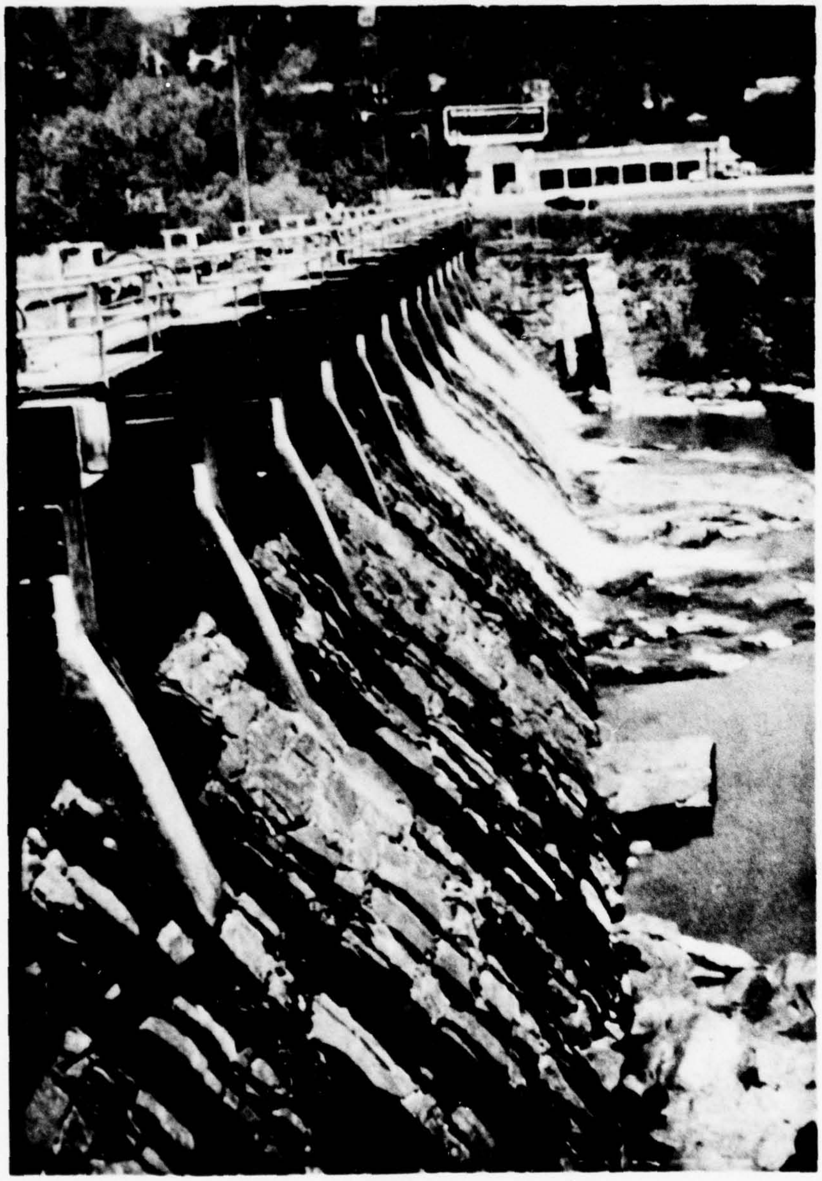
Douglas L. Haller
Colonel, Corps of Engineers
District Engineer

Recommended By:

ORIGINAL SIGNED BY:
CARL S. ANDERSON, JR.

for Jack G.Starr, R.A., P.E.
Chief, Engineering Division

Date: AUG 24 1979



OVERVIEW PHOTO

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
SMITH RIVER DAM

SECTION 1 - PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2. Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (See Reference 1, Appendix V). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Smith River Dam is a stone masonry gravity dam approximately 538 ft long. The principal spillway consists of a 322 ft long gated overflow section at elevation 690 msl (top of concrete sill). The principal spillway gates are 6 ft high steel gates which lift upward to allow water to pass over the principal spillway. The auxiliary spillway is 144.5 ft long with 3 ft high timber flash boards located on top of the dam. The crest of the flashboards is at elevation 697.5 msl. The powerhouse intake structure consists of two 15 by 16 ft screened openings with a crest elevation of 684. The powerhouse and intake structure occupy 56 ft of the

71.5 ft of non overflow section of the dam (see plan, Sheet 2, Appendix I). The non overflow section crest is at elevation 703 msl.

1.2.2 Location: Smith River Dam is located on the Smith River near its juncture with U.S. Route 58-220 at Martinsville, Virginia (See Sheet 1, Appendix I).

1.2.3 Size Classification: The dam is classified as an "intermediate" size structure because of the maximum storage capacity of 2600 acre-feet at the top of the dam (elev. 703).

1.2.4 Hazard Classification: The dam is located in a surburban area, and based upon the downstream proximity of industrial and commerical development, U.S. Route 58-220 and several homes, the dam is assigned a "high" hazard classification. The hazard classification used to categorize a dam is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: The City of Martinsville is the owner of the dam.

1.2.6 Purpose: The Smith River Dam was built for electrical power production for the City of Martinsville.

1.2.7 Design and Construction History: The dam was designed by Lockwood Greene, Inc., Boston, Mass., in 1904, under the supervision of the City of Martinsville. The original contractor is unknown. The original powerhouse was located to the left of the dam below the auxiliary spillway. This powerhouse was abandoned and the existing powerhouse was built at the right end of the dam and the principal spillway

gates were installed in 1931. Design of the powerhouse relocation, principal spillway gates and flash boards on the auxiliary spillway was by Saville and Williamson, Inc., Richmond, Virginia.

1.2.8 Normal Operational Procedures: Flow to the Smith River Dam is partially regulated by Philpott Reservoir which is approximately 20 miles upstream. The principal spillway gates are normally closed (down) and river flow is directed through the powerhouse intake. When water is released upstream at Philpott Reservoir, the principal spillway gates are opened to maintain a reservoir level approximately 5 ft above the principal spillway crest (elevation 695 msl). Normal pool elevation varies from 694 to 697.4 msl, and is affected by flows released from Philpott Reservoir.

1.3 Pertinent Data:

1.3.1 Drainage Areas: The total drainage area for the Smith River at the dam site is $380^{\pm}$ square miles of which 212 square miles are regulated by Philpott Reservoir located approximately 20 miles upstream. The remaining 168 sq. miles of drainage area flows directly into the Smith River reservoir.

1.3.2 Discharge at Dam Site: Maximum known flood at the dam site occurred in 1937 when a flow of 39,000 cfs was recorded by the USGS gaging station (02073000) located just downstream of the dam.

Powerhouse Intake Discharges:

Pool Elevation at Principal Spillway Crest (690 msl)	705 CFS
Pool Elevation at Top of Principal Spillway Gates (696 msl)	1995 CFS

Principal Spillway Discharge:

Crest Gates Up and Pool Elevation at bottom of gates (696 msl) 13,583 CFS

1.3.3 Dam and Reservoir Data: See Table 1.1, below.

Table 1.1

Item	Elevation feet msl	Area Acres	Reservoir Capacity Acre Feet	Reservoir Capacity Watershed Inches(a)	Length Miles
Top of Dam	703	368	2600	0.3	5.0
Top of Flashboards	697.5	176	1015	0.1	2.4
Top of Principal Spillway Gates	696	54	800	0.09	1.0
Crest of Principal Spillway	690	40	500	0.06	0.8
Streambed at Centerline of Dam	665	-	-	-	-

(a) Based on 168 Sq. Mi.

SECTION 2 - ENGINEERING DATA

2.1 Design: No record of the original design is available. However, the City of Martinsville has records of gaugings taken at the site in 1904 by Lockwood Greene and Company (architects and engineers) and it is assumed that this company completed the original design of the dam. The original impoundment was designed as a hydroelectric plant with a spillway race and powerhouse located downstream from the left abutment. In 1931 the firm of Saville and Williamson, Inc., (Richmond, Virginia) prepared construction plans which added gates to the principal spillway and flash boards to the auxiliary spillway, and relocated the powerhouse to its present location in the right abutment.

2.2 Construction: The construction records were not available.

2.3 Operation: Documentation of procedures concerning the operation of the powerhouse intake and spillway gates were not available, however the City of Martinsville has a standard operation procedure used by the dam operator.

2.4 Evaluation: Original design data is non-existant, however, data of repairs and improvements are sufficiently documented by drawings. Operational procedures are adequate.

SECTION 3 - VISUAL INSPECTION

3.1 General: An inspection of Smith River Dam was made 1 May 1979 and the pool was at elevation 694.5 msl. The weather was fair and the temperature was 72^{O}F.

3.2 Dam and Appurtenances:

3.2.1 Findings: At the time of inspection the dam was in good condition. Field observations are outlined in Appendix III. Seepage was observed at scattered locations along the downstream slope of the left abutment and along the left auxiliary spillway at Pier No. 4. Seepage was also located on the face of the auxiliary spillway below the flash boards between Piers No. 2 and 3 and through the planking which blocks former gates between the right abutment of the auxiliary spillway and Piers No. 1 and 2. Seepage identified at each of these areas was flowing at an estimated rate of less than 1 gpm. The area behind the auxiliary spillway from which seepage is occurring is completely silted in above the level of the seepage.

Considerable leakage was passing under and around the seals of the crest gates, particularly for Gates No. 7 through 14 numbered from the right abutment. The seepage flow ranged from less than 1 gpm to more than 100 gpm (Gates No. 8 and 11). The structure surface shows no apparent seepage along the face below the level of the gates. However, seepage was noted at the right abutment just downstream from the building.

Numerous bedrock outcrops were exposed in the downstream river channel at the toe of the dam at the time of the in-

spection. A smaller number of outcrops are exposed in the abutments. The Leatherwood Granite is exposed in the left abutment and throughout much of the river bed. This rock appears to be a slightly to moderately weathered, light gray to brown, fine to coarse-grained granite. Amphibolite or biotite gneiss and quartz veins were observed locally. A rather well-defined rectangular joint system is exhibited in the granite. Measured joint sets have strikes of approximately 25 degrees to the northeast and 55 degrees to the northwest. The joint surfaces dip from near vertical to vertical. Scattered oblique joints were also encountered. The depth of water prevented the examination of all outcrops, consequently the gneiss-granite contact was not observed. Geologic literature describes this contact as being rather sharp; however inclusion of the surrounding country rock or gneiss would be expected in the granite during its implacement. This would explain the presence of amphibiotite or biotite gneiss in the granite. Biotite gneiss is exposed at scattered intervals in the slopes bounding the road to the powerhouse. No faults were observed in the field during this investigation and geologic maps of the area do not show the presence of any faults in the immediate vicinity.

The dam appears to be founded on bedrock. At the high points of the rock outcrops along the downstream toe, the dam appears embedded but this could not be verified.

The grout in the rubble surface was found to be in good condition except on the downstream face of the auxiliary spillway where the old powerhouse race was built. The grout had severely eroded from the rock surface, and there was leakage where the original head gates were plugged. The vertical and horizontal alignment appeared to be good. It was noted that one principal spillway gate is inoperable at the time of inspection. The slurry gate located in the powerhouse intake is also inoperable at this time.

3.2.2 Reservoir: The reservoir has side slopes of approximately 2:1 and are wooded. A sediment buildup was measured on both ends of structure, particularly the left, and where the silt buildup is at the top of the dam adjacent to the original powerhouse gates. No debris was noted upstream. The daily surge from Philpott appears to bring debris to the intake structure where it is cleaned by a trash rack. No sloughing of bank slopes was observed.

3.2.3 Downstream Area: The stream is stable with a rock bottom showing essentially no signs of erosion or scouring. A highway bridge crosses the river some 100 ft to 150 ft downstream. The bridge decking is at the approximate height of the top of impoundment. The downstream slopes are approximately 2:1 and the stream width is constant for some distance downstream. The only debris noted downstream were rocks, as can be seen in pictures in Appendix II. It was also noted

that there are several homes, industries, a sewage treatment plant, and shopping center located downstream.

3.3 Evaluation:

3.3.1 Dam and Abutments: In general, the structure appears to be in good condition. It is recommended that all the principal spillway gates be put in good working condition and that the concrete sills be repaired. All areas subject to corrosion should be painted. Deterioration of the masonry in the left auxiliary spillway should be repaired. The seepage in the left auxiliary spillway area should be monitored quarterly to detect any changes in flow rate.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: The Smith River Dam is used for electrical power production. The normal pool elevation is maintained by the powerhouse intake and gates on the principal spillway. The powerhouse intake controls water passing through the dam and maintains a pool elevation of approximately 694 to 696.5. The pool must be maintained at a minimum elevation of 694 due to upstream industrial water intake structures. During periods (daily) when water is reléased from Philpott Reservoir the gates are opened during large increases in inflows which cannot be controlled by the powerhouse intake in order to maintain the pool elevation below 697 msl.

4.2 Maintenance of Dam and Appurtenances: The maintenance is the responsibility of the City of Martinsville. Dam maintenance consists of the following:

1) Inspection of dam and gates each 8 hour shift (3 shifts per day)
2) Removal of debris from powerhouse intake several times during each 8 hour shift.
3) Repair of appurtenances on an as needed basis.
4) Daily check of operation of principal spillway gates.

4.3 Warning System: No warning system exists.

4.4 Evaluation: The dam and appurtenances are in good operating condition, and maintenance is being routinely performed.

SECTION 5 - HYDRAULICS/HYDROLOGIC DESIGN

5.1 Design: No data was available for the Smith River Dam.

5.2 Hydrologic Records: A USGS gaging station (020730) is located about ½ mile downstream of the dam with a drainage area of 380 sq. miles. This gage has been in operation since 1930 and recorded a maximum river flow of 39,000 cfs in October 1937.

5.3 Flood Experience: The maximum pool elevation observed was reached during a storm in June, 1972 and the flow recorded on the USGS gage downstream was 20,900 cfs. The pool rose to elevation 698 msl during the tropical storm Agnes rainfall in June 1972. There were no records of pool elevation in the 1937 flood.

5.4 Flood Potential: In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that maybe expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. The Probable Maximum Flood (PMF), and ½ PMF hydrographs were developed by the SCS method (Reference 4, Appendix V). Precipation amounts for the flood hydrographs of the PMF and ½ PMF were taken from the U. S. Weather Bureau information (Reference 5, Appendix V). Appropriate adjustments for basin size and shape were accounted for, and flood hydrographs were determined by utilizing unit hydrographs of the Smith River Basin as provided by the Wilmington

District Corps of Engineers. These hydrographs were routed through the spillway to determine maximum pool elevations.

5.5 Reservoir Regulation: The reservoir is regulated by the principal spillway gates and water released from Philpott Reservoir. Philpott Reservoir reduces the inflow to the dam significantly by delaying peak flows and effectively reducing flow rates to the dam by better than 50%.

For routing purposes the pool elevation at the beginning of the flood was assumed to be 695 msl with the principal spillway gates down and the powerhouse intake was not utilized. Reservoir stage-storage data were taken from the USGS Quadrangle (Martinsville West) and stage-discharge data was computed from available plans.

Floods were routed through Philpott Reservoir and added to the local inflows at Martinsville to determine the inflow hydrograph with time lag of 6 hours.

5.6 Overtopping Potential: The predicted rise of the reservoir pool and other pertinent data were determined by routing the flood hydrographs through the reservoir as previously described. The results for the three flood conditions (PMF and ½ PMF) are shown in the following Table 5.1.

TABLE 5.1 RESERVOIR PERFORMANCE

	Normal Flow	Flood Hydrograph ½ PMF	Flood Hydrograph PMF
Peak Flow, CFS			
Inflow	15±	62,568	141,443
Outflow	8±	62,945	141,950
Maximum Elev., ft msl		708.1	716.2
Non Overflow Section (Elev. 703 msl)			
Depth, ft		5.1	13.2
Duration, hrs		16	24
Velocity, fps*		9.7	15.5
Top of Principal Spillway Gates (Gates Down) (Elev. 696 msl)			
Depth, ft		12.1	20.25
Duration, days		3-4	3-4
Velocity, fps*		16	20.7
Tailwater Elev. (ft msl)	664	694	717

*Critical Velocity at Control Section

5.7 Reservoir Emptying Potential: The reservoir cannot be lowered below elevation 684 msl. During a normal dry period, a river flow of 200 cfs is expected and it would take approximately 8 hours to lower the reservoir to elevation 684 msl.

5.8 Evaluation: The appropriate spillway design flood is the PMF due to the "high" hazard classification. The spillway will pass 18% of the PMF prior to the reservoir overtopping the non overflow section (elevation 703 msl). Flood routing calculations indicate that overtopping of the dam will occur and pool height above the non overflow section will be 12.1 ft for the PMF and 5.1 ft for the ½ PMF.

The hydrologic data used in the evaluation pertains to present day conditions with no consideration given to future development.

SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: Smith River Dam is believed to be founded across or immediately beside a contact between the Leatherwood Granite and biotite gneiss bedrock. Other than general improvement drawings (1931-1932), the City of Martinsville was unable to locate the contract drawings or design and construction data related to the initial construction of the structure in the early 1900's. Consequently, only generalized information obtained from the improvement drawings and by visual inspection is reported.

The dam site is located within the Piedmont Physiographic Province of Virginia, which is underlain primarily by metamorphic and igneous rocks, but also includes some sedimentary rocks. These rocks range from Precambrian to Triassic in age (Reference 3, Appendix V).

The dam site appears to be underlain by biotite gneiss (Precambrian to Cambrian Age) and the Leatherwood Granite (Late Paleozoic Age) bedrock. The biotite gneiss is a banded, light to dark gray colored rock which is fine to coarse-grained and is foliated (includes parallel orientation of minerals). It is exposed in the right abutment slopes. The gneiss was intruded in the geologic past by the younger Leatherwood Granite, an igneous rock. The Leatherwood is a light gray coarse-grained, porphyritic granite and is exposed in the riverbed and at scattered locations along the left abutment. Foliation previously measured in nearby biotite gneiss outcrops strikes approximately 35 to 60 degrees to the northeast and dips 45 to 80 degrees to the northwest. The thickness of these rock units is not known.

The fact that the structure may rest on the contact between two different rock types, in this case, has very little bearing upon the structural integrity of the dam. The intrusion of the granite into the pre-existing biotite gneiss was done under considerable heat and pressure. The associated contact metamorphism process created a gradational bond between the two rock types, which should be no more susceptible to weathering and weakening than within the individual rock types themselves.

6.2 Evaluation:

6.2.1 Foundation and Abutments: The improvement drawings included in Appendix I do not include any sections showing the foundation of the structure. Sheet 4, Appendix I includes a section listing rock at the bottom of Piers No. 2, 3, 4 and 5; ranging from elevation 681.5± (No. 2) to elevation 692.0± (Nos. 4 and 5). Rock is also exposed along the downstream toe of the dam (See Overview Photo). Based upon this information and the abundance of bedrock in the downstream channel, it is assumed the dam is founded on bedrock, but we cannot verify that the structure is keyed into the bedrock. Assuming a foundation on biotite gneiss and/or granite, excessive settlement of the dam does not appear to be a problem since outcrops in the immediate area consist of fairly competent, slightly to moderately weathered bedrock. Measured attitudes indicate there are probably no adversely oriented weak planes within the foundation rock that would act as a potential sliding plane. It is not known whether a cutoff trench was installed during construction; however, only minor seepage would be

expected along joint patterns in the foundation rock, if it is similar to that observed in the downstream channel. The tail water height in the downstream channel prevented the observation of any seepage under the dam.

The right abutment intersects a very steep natural slope which includes scattered biotite gneiss outcrops. The abutment appears to tie into slightly to moderately weathered biotite gneiss at the base, but may include some residual soil up higher. The abutment slope appeared stable with the exception of minor sloughing along the roadway, which occurs in cut areas. The slope above the sloughing areas is thickly wooded.

Bedrock is exposed along the base of the auxiliary spillway in the left abutment. Examination of the physical setting at the left abutment suggests that the dam rests upon or ties into bedrock.

6.2.2 Stability Analysis: An accurate stability analysis could not be made since neither contract drawings nor construction records are available to indicate the foundation embedment. However, an evaluation was made in accordance with Section 4.4 of Reference 1, Appendix V. Assuming the structure resting upon a horizontal bedrock surface, the stability was evaluated with respect to sliding resistance and overturning assuming water at the dam crest, and 22 ft over the crest which corresponds to the PMF. Calculations are included in Appendix IV. Factors of safety of 1.34 and 7.42 were obtained for the sliding conditions for the two pool level conditions, respectively. The factor of safety

for sliding under normal pool is substantially less than the factor of safety of 3 required by Reference 1, Appendix V. Since the visual inspection revealed some apparent embedment into the rock foundation at high points along the downstream toe, we believe the dam is considerably safer than indicated by the analysis.

The stability of the structure with respect to overturning for the two reservoir conditions was also determined. At normal pool, the resultant of all forces passes within the 2.6 ft of the toe of dam and this is beyond the middle third. Under surcharged conditions caused by the PMF, the resultant of all forces is within the middle third of the dam base which is acceptable. Without direct evidence in the form of design drawings that the dam is keyed into the bedrock, and assuming only a surface of bedrock foundation, the dam appears to be marginal both with respect to sliding resistance and to overturning under normal pool loading. This computation does not consider end restraint and assumes that the uplift pressures below the base of the dam are equal to the water head behind the dam at the heal. It is not likely that actual pressures are so high.

Since failure is not indicated and the dam has withstood 47 years of service under present loading conditions, we do not believe additional study is necessary.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: The Smith River dam at the time of inspection appeared sound and in a safe operating condition. The dam will be overtopped by the ½ PMF and PMF; however, the structure is not expected to fail. Based

on the visual inspection, there are no apparent problems which require immediate action for the normal pool conditions.

7.2 Remedial Measures: There is no immediate need for remedial measures; however, the following repair, maintenance and monitoring measures are suggested.

7.2.1 Seepage occurring in the plugged openings of the auxiliary spillway should be monitored quarterly. Increased flow rates may required corrective measures to insure the dam's integrity.

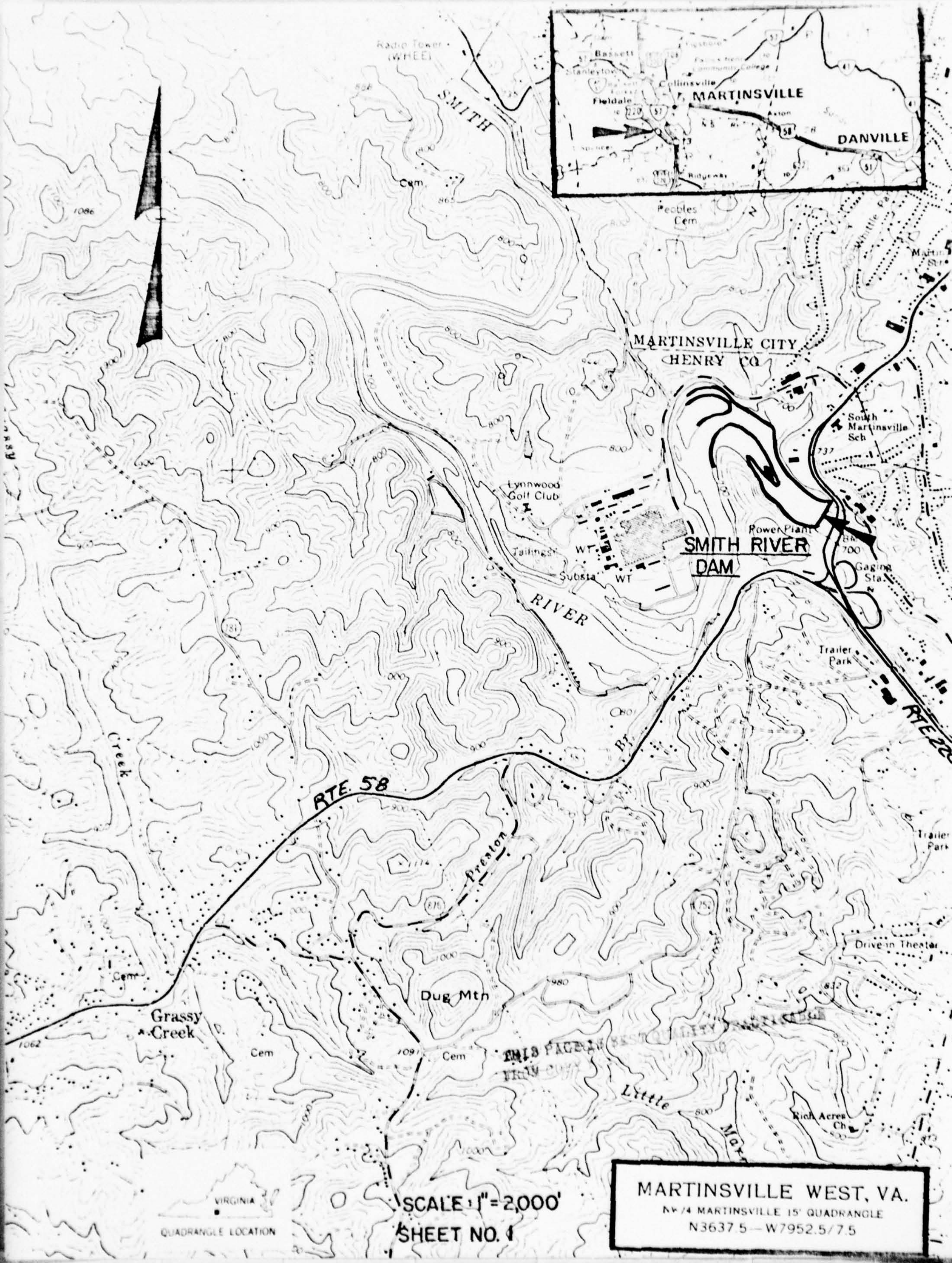
7.2.2 Deterioration of the stone masonry should be corrected along the downstream face of the dam at the auxiliary spillway.

7.2.3 The concrete sill along the spillway crest should be repaired to minimize leakage through the crest gates and protect the crest from further deterioration.

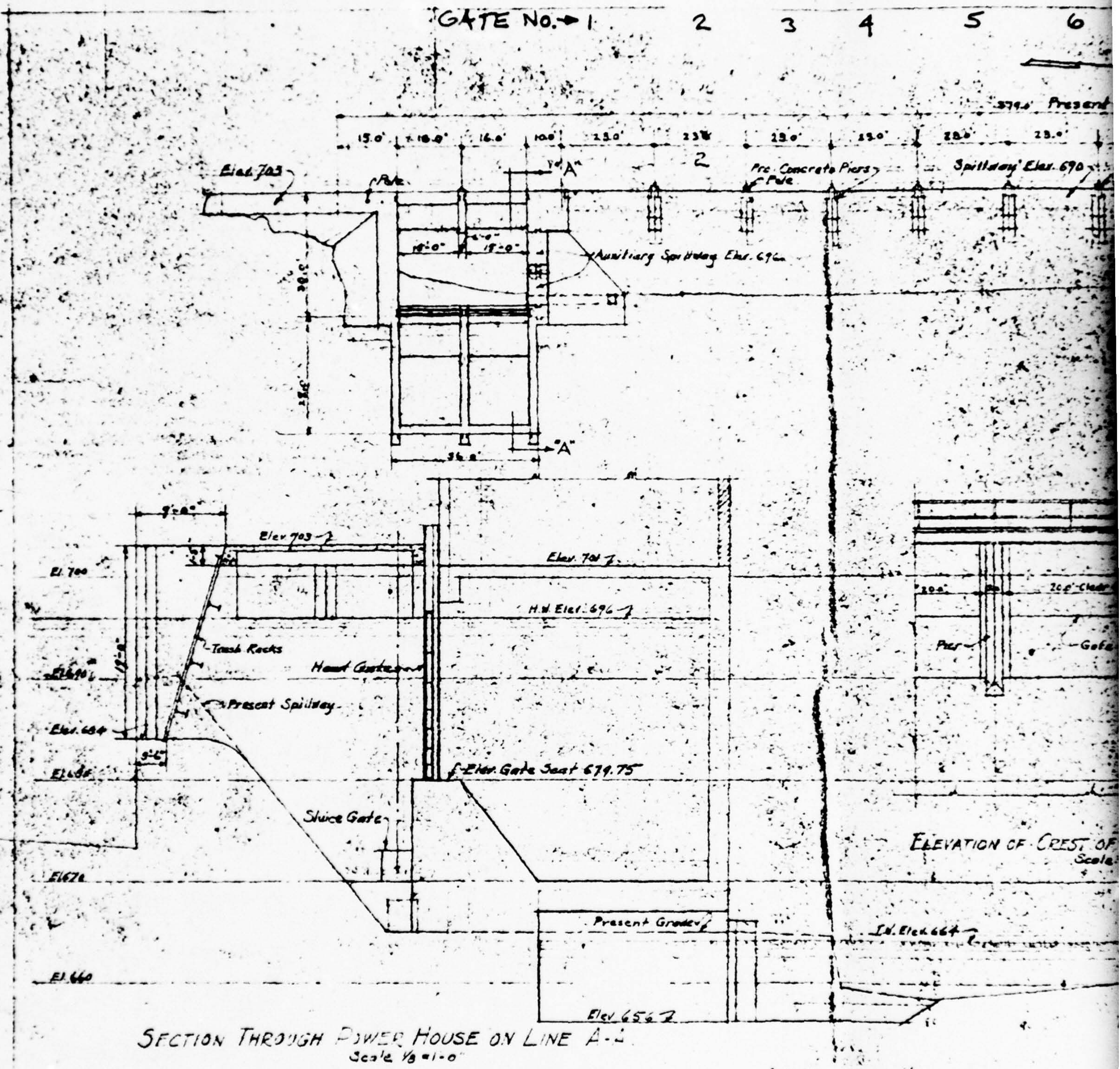
7.2.4 A staff gage should be installed to monitor high water flows.

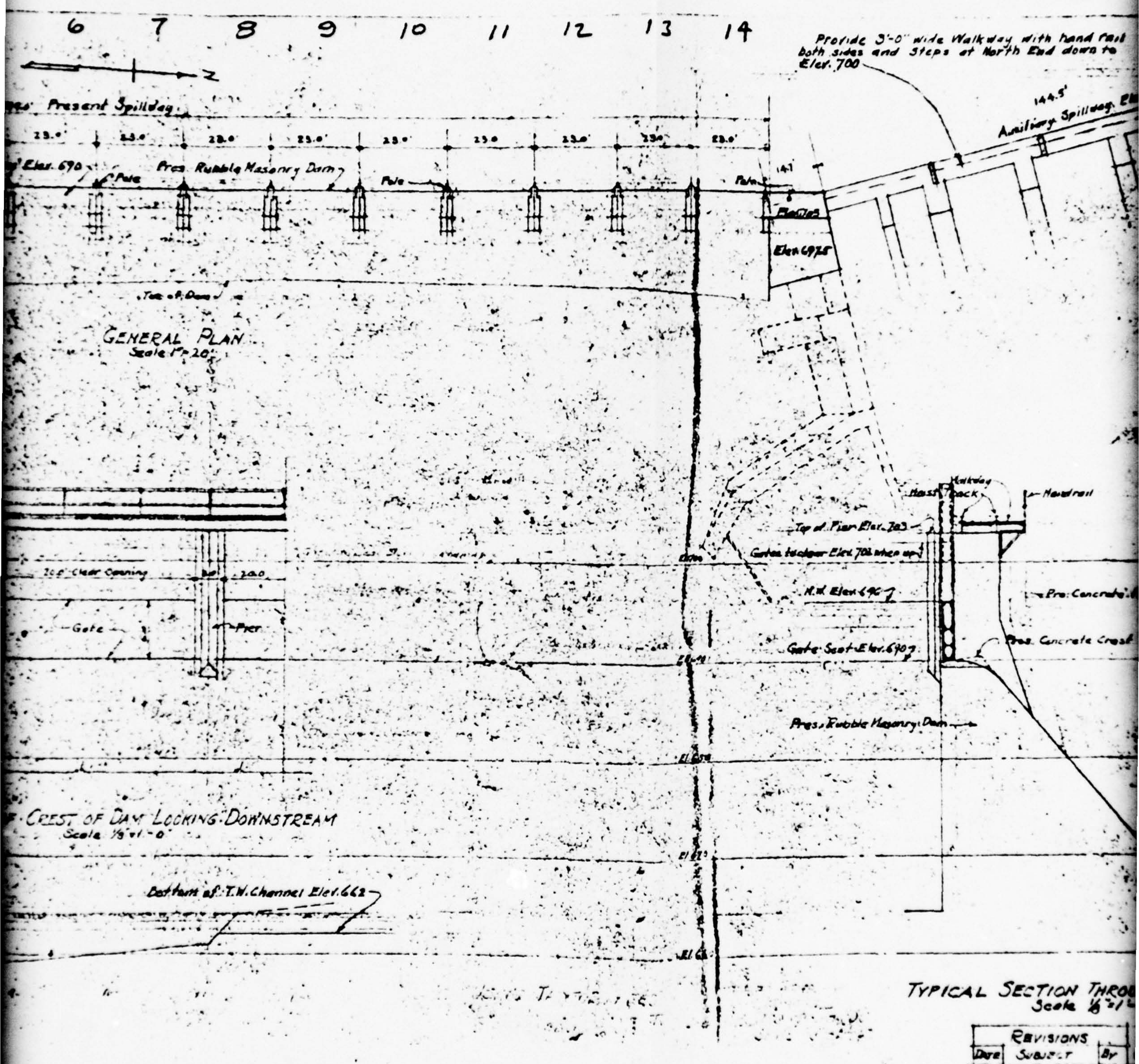
APPENDIX I

MAPS AND DRAWINGS



MARTINSVILLE
DANVILLE
Bassett
Stanleytown
Collinsville
Fieldale
Ridgeway
Radio Tower (WHEE)
SMITH
Cem
Peebles Cem
MARTINSVILLE CITY
HENRY CO
South Martinsville Sch
Lynnwood Golf Club
Tailings
WT
Substa
Power Plant
SMITH RIVER DAM
Gaging Sta
RIVER
Trailer Park
RTE 220
RTE 58
Creek
Preston
Br
Drive-in Theater
Dug Mtn
Grassy Creek
Little
Rich Acres Ch
VIRGINIA
QUADRANGLE LOCATION
SCALE: 1" = 2,000'
SHEET NO. 1
MARTINSVILLE WEST, VA.
NW/4 MARTINSVILLE 15' QUADRANGLE
N3637.5—W7952.5/7.5

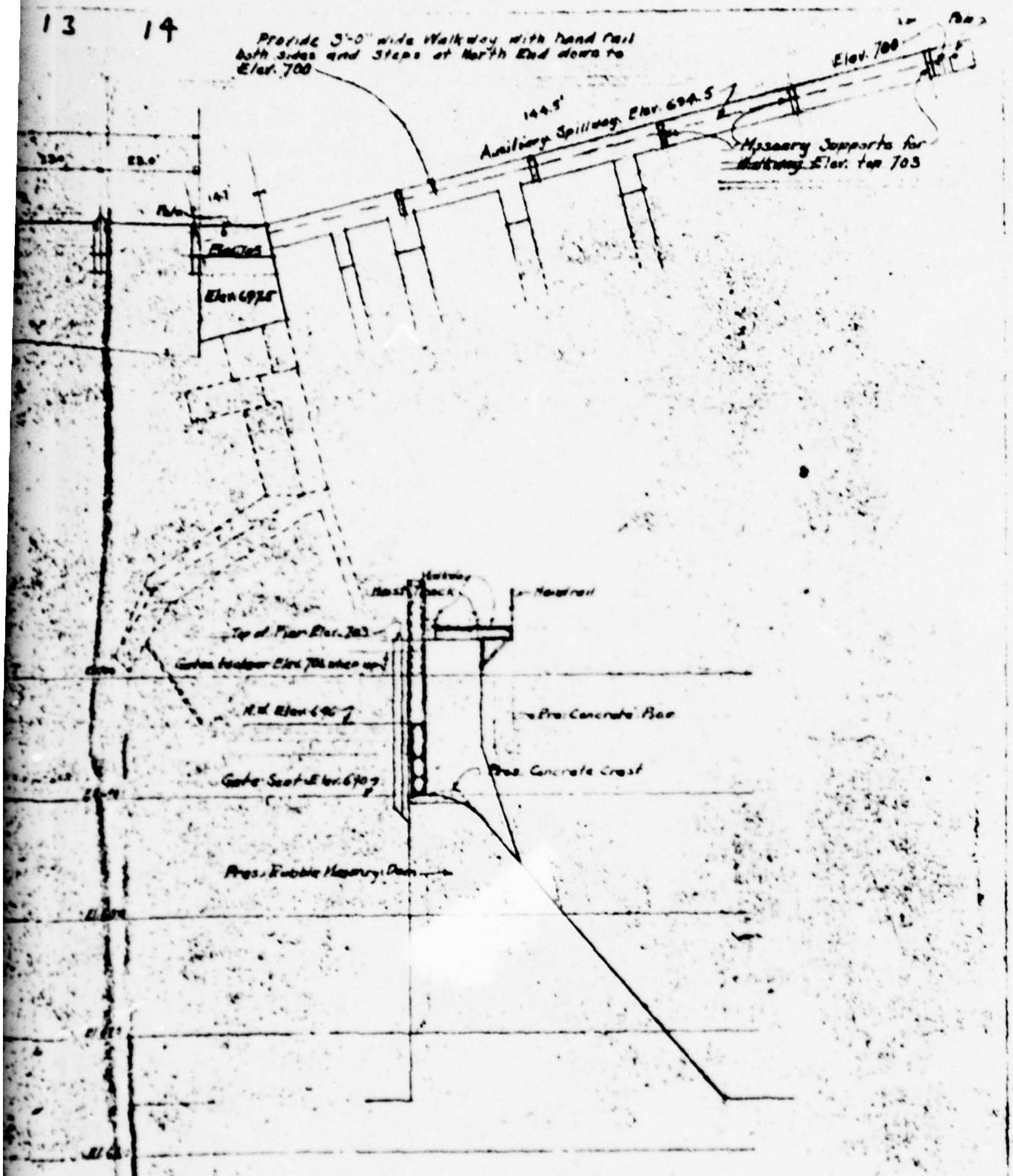

GATE NO. 1
2
3
4
5
6
Present
15.0'
10.0'
16.0'
10.0'
23.0'
23.0'
23.0'
23.0'
23.0'
23.0'
Elev. 703
Pole
"A"
Pre. Concrete Piers
Pole
Spillway Elev. 690
15'-0"
15'-0"
Auxiliary Spillway Elev. 696
"A"
Elev. 703
Elev. 701
El. 700
H.W. Elev. 696
Trash Racks
Head Gates
Present Spillway
Elev. 684
Elev. Gate Seat 679.75
Sluice Gate
El. 660
Present Grade
Inv. Elev. 664
Elev. 656
Pier
ELEVATION OF CREST OF
Scale
SECTION THROUGH POWER HOUSE ON LINE A-A
Scale 1/8" = 1'-0"



6 7 8 9 10 11 12 13 14
N
Present Spillway
23.0'
Elev. 690
Pole
Pres. Rubble Masonry Dam
Top of Dam
GENERAL PLAN
Scale 1" = 20'
Provide 3'-0" wide Walkway with hand rail both sides and Steps at North End down to Elev. 700
144.5'
Auxiliary Spillway
Elev. 697.5
Walkway
Hoist Track
Handrail
Top of Pier Elev. 703
Gates locked Elev. 702 when up
H.W. Elev. 690
Pres. Concrete
Gate Seat Elev. 690
Pres. Concrete Crest
Pres. Rubble Masonry Dam
20.5' Clear Opening
20.0
Gate
Pier
CREST OF DAM LOOKING DOWNSTREAM
Scale 1/8" = 1'-0"
Bottom of T.W. Channel Elev. 662
TYPICAL SECTION THRO
Scale 1/8" = 1'
REVISIONS
Date
Subject
By

2

13 14

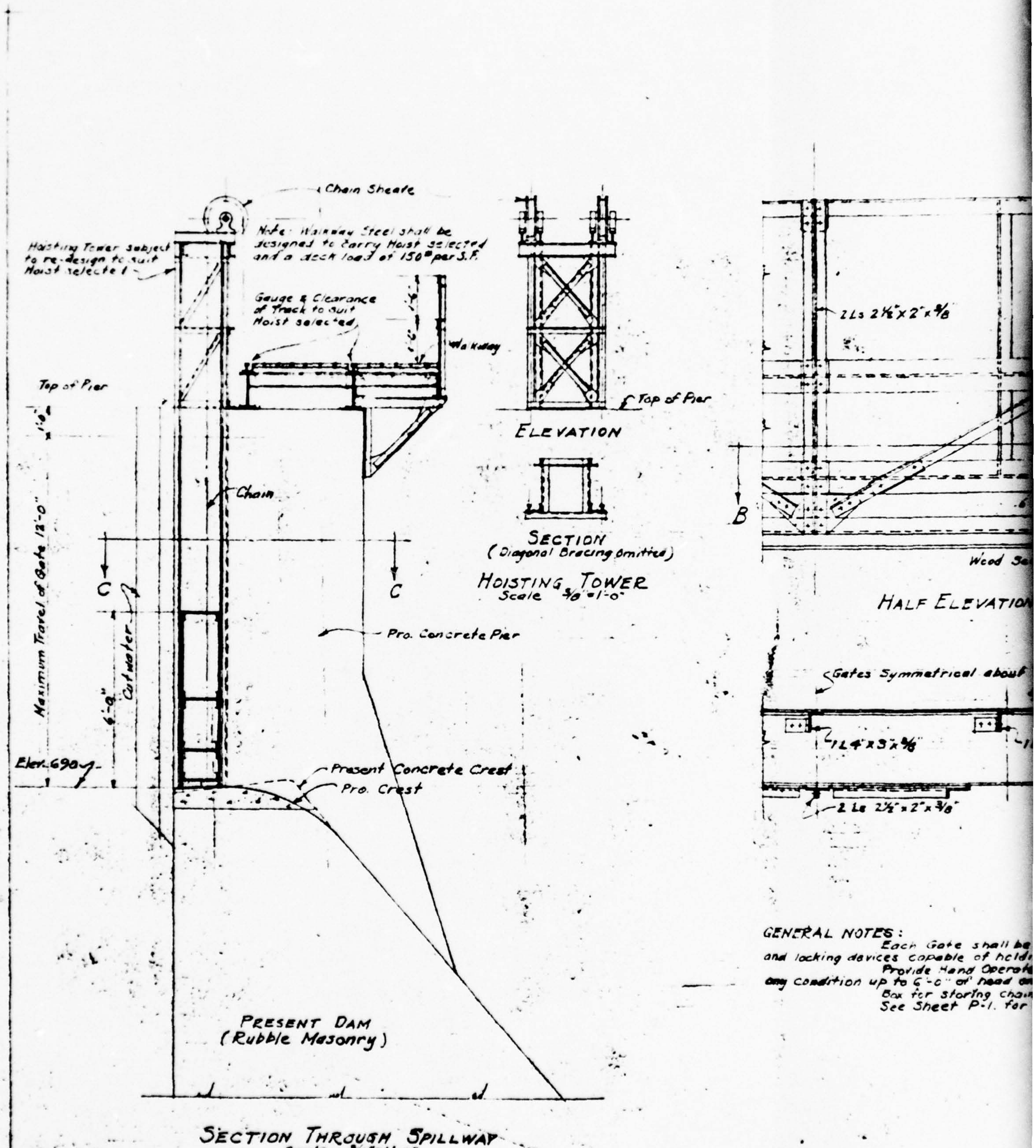


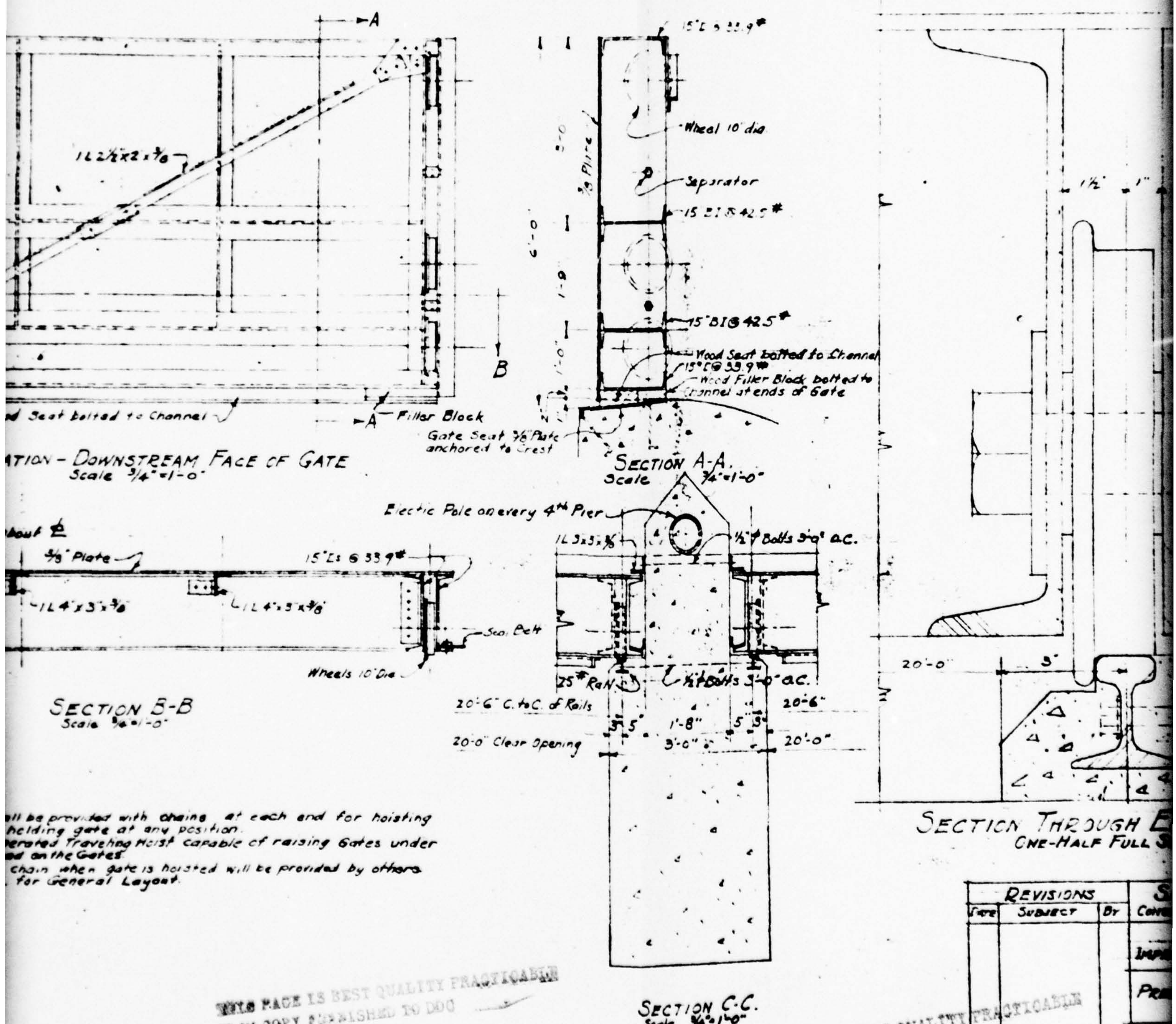
TYPICAL SECTION THROUGH SPILLWAY
Scale 1/8"=1'-0"

SHEET 2

3

REVISIONS			SAVILLE AND WILLIAMSON, INC. CONSULTING ENGINEERS RICHMOND, VA.
Date	Subject	By	MARTINSVILLE, VA. IMPROVEMENTS TO HYDRO-ELECTRIC PLANT
			SKETCH SHOWING GENERAL LAYOUT OF CREST GATES
			Scale As noted; Drawing No. A-106; Sheet No. P-1


Chain Sheave
Note: Walkway Steel shall be designed to carry Hoist selected and a deck load of 150# per S.F.
Hoisting Tower subject to re-design to suit Hoist selected
Gauge & Clearance of Track to suit Hoist selected
Walkway
Top of Pier
Maximum Travel of Gate 12'-0"
Chain
C
C
Cutwater
6'-0"
Pro. Concrete Pier
Elev. 698
Present Concrete Crest
Pro. Crest
PRESENT DAM
(Rubble Masonry)
SECTION THROUGH SPILLWAY
Scale 3/8"=1'-0"
ELEVATION
Top of Pier
SECTION
(Diagonal Bracing omitted)
HOISTING TOWER
Scale 3/8"=1'-0"
2Ls 2½"x2"x3/8"
B
Wood Se
HALF ELEVATION
Gates Symmetrical about
1L4"x3"x3/8"
2 Ls 2½"x2"x3/8"
GENERAL NOTES:
Each Gate shall be
and locking devices capable of holdi
Provide Hand Operate
any condition up to 6'-0" of head on
Box for storing chain
See Sheet P-1. for



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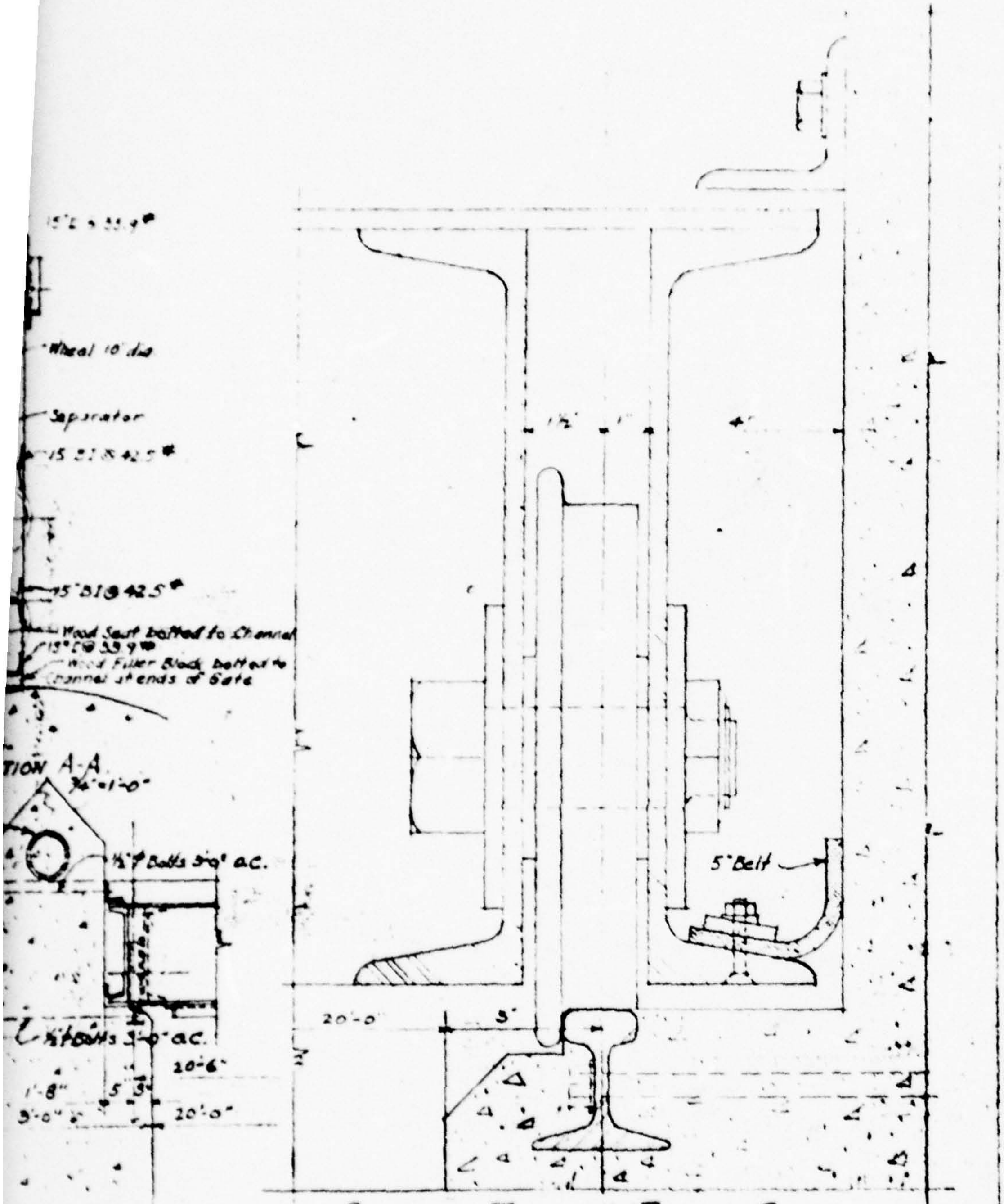


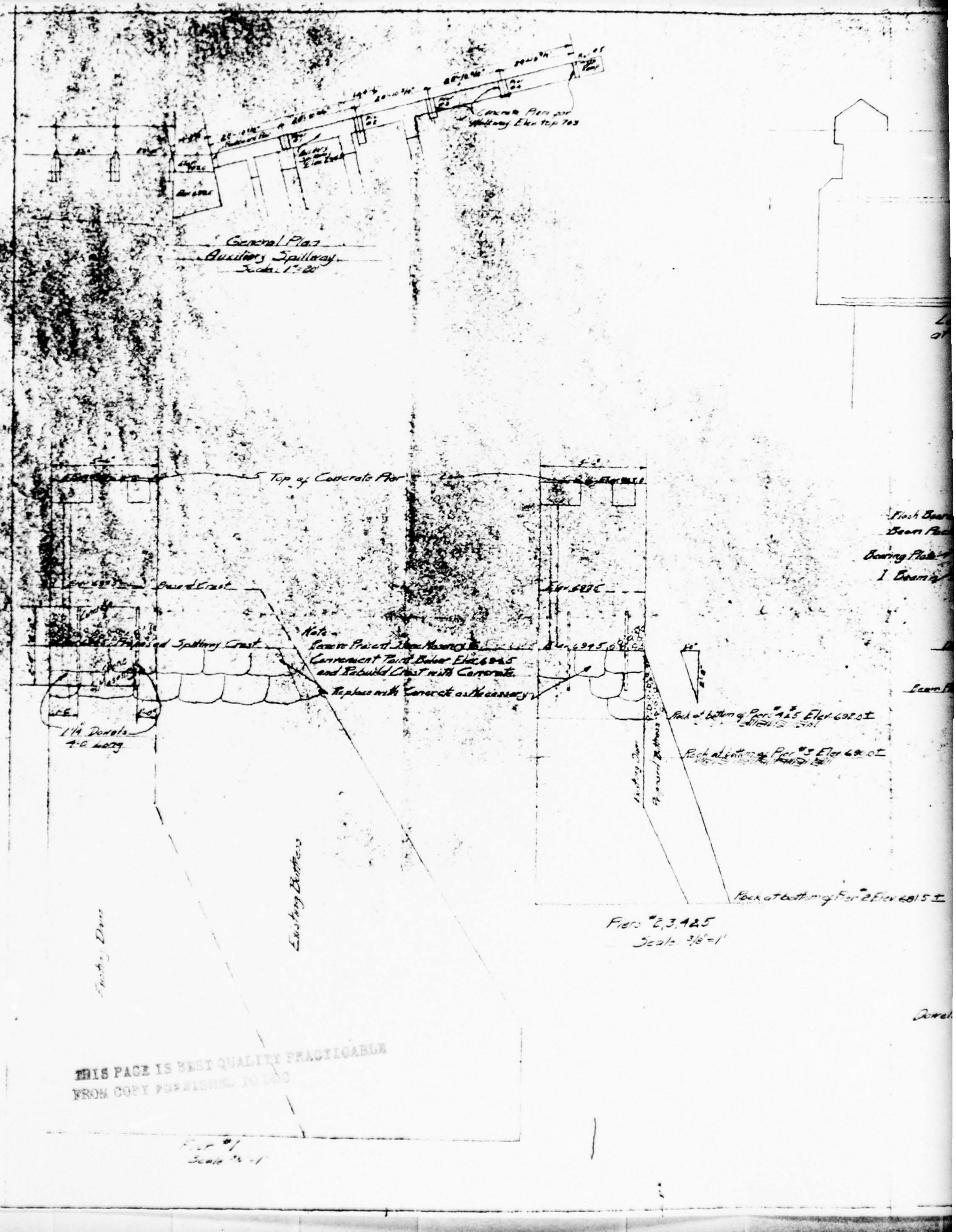
SECTION THROUGH END OF GATE
ONE-HALF FULL SIZE

3

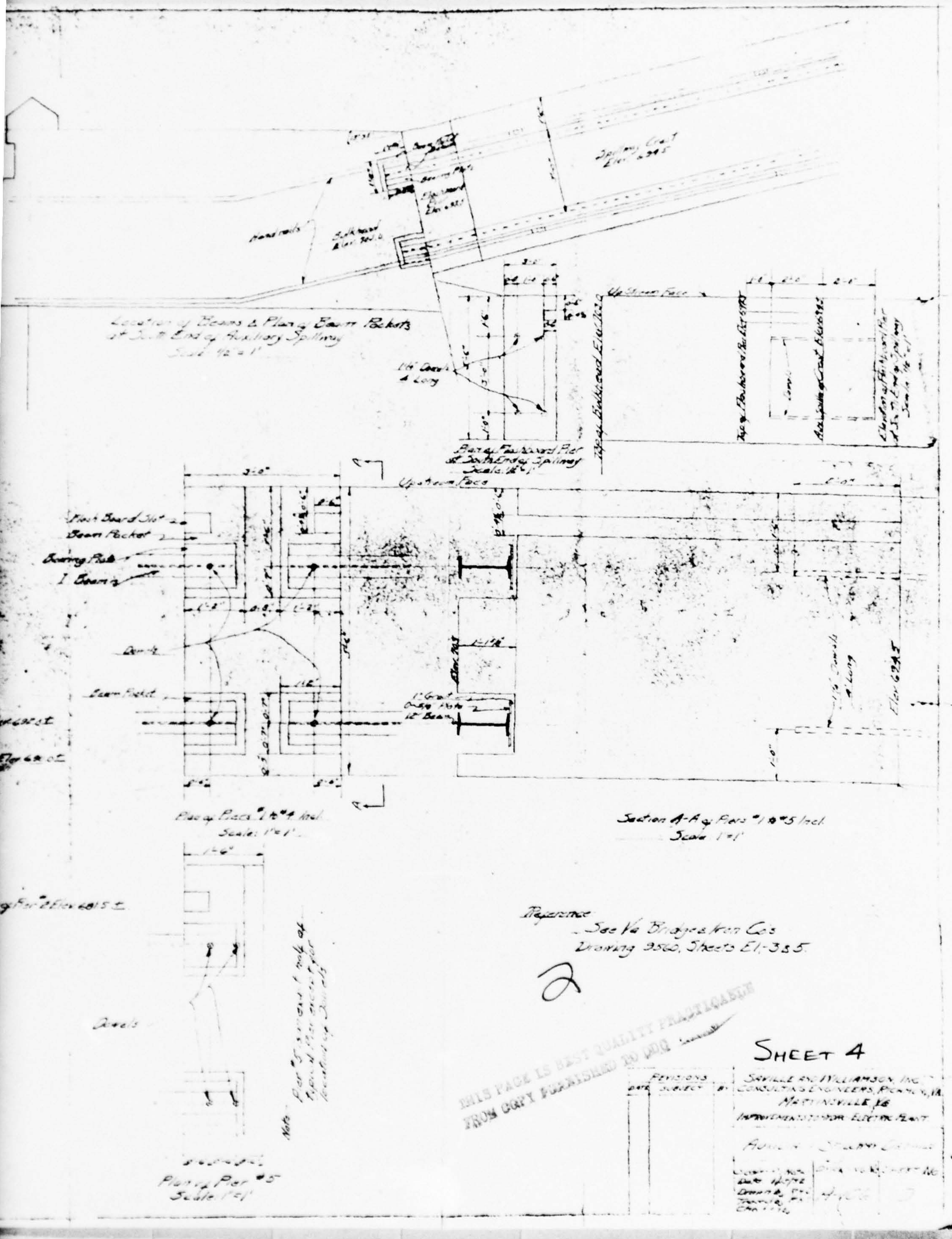
SHEET 3

ECTION C.C.

REVISIONS			SAVILLE AND WILLIAMSON, INC. CONSULTING ENGINEERS RICHMOND, VA.	
Date	Subject	By	MARTINSVILLE, VA. IMPROVEMENTS TO HYDRO-ELECTRIC PLANT	
			PRELIMINARY DETAILS OF CREST GATES FOR CHAIN OPERATION	
			Scale As Noted Date 7-14-31 Drawn by HSC Traced by HSC Checked by	DRAWING No. A-106. / SHEET No. P-2.



General Plan
Auxiliary Spillway
Scale: 1"=20'
Concrete Piers for Walkway Elev. top 703
Top of Concrete Pier
Board Crest
Proposed Spillway Crest
Note -
Remove Present Stone Masonry & Cement Paint Below Elev. 694.5 and Rebuild Crest with Concrete.
Replace with Concrete as Necessary
1¼" Dowels 4-0 Long
Existing Buttress
Existing Dam
Rock at bottom of Piers #4 & 5 Elev. 692.0 ±
Rock at bottom of Pier #3 Elev. 690.0 ±
Rock at bottom of Pier #2 Elev. 681.5 ±
Piers #2, 3, 4 & 5
Scale 3/8"=1'
Pier #1
Scale 3/8"=1'
Bearing Plate
I Beam
Dowel



Location of Beams & Plan of Beam Pockets at South End of Auxiliary Spillway
Scale: 1/2" = 1'
Handrails
Spillway Crest
Bulkhead
Plan of Flashboard Pier at South End of Spillway
Scale: 1/2" = 1'
Upstream Face
Flash Board Slot
Beam Pocket
Bearing Plate
I Beams
Dowels
Plan of Piers #1 to #4 Incl.
Scale: 1" = 1'
Section A-A of Piers #1 to #5 Incl.
Scale: 1" = 1'
Plan of Pier #5
Scale: 1" = 1'
Reference
See Va. Bridge & Iron Co's
Drawing 9560, Sheets E1-3 & 5.
2
Sheet 4
Revisions
Date
Subject
By
Saville and Williamson, Inc.
Consulting Engineers
Martinsville, Va.
Improvements to Electric Plant

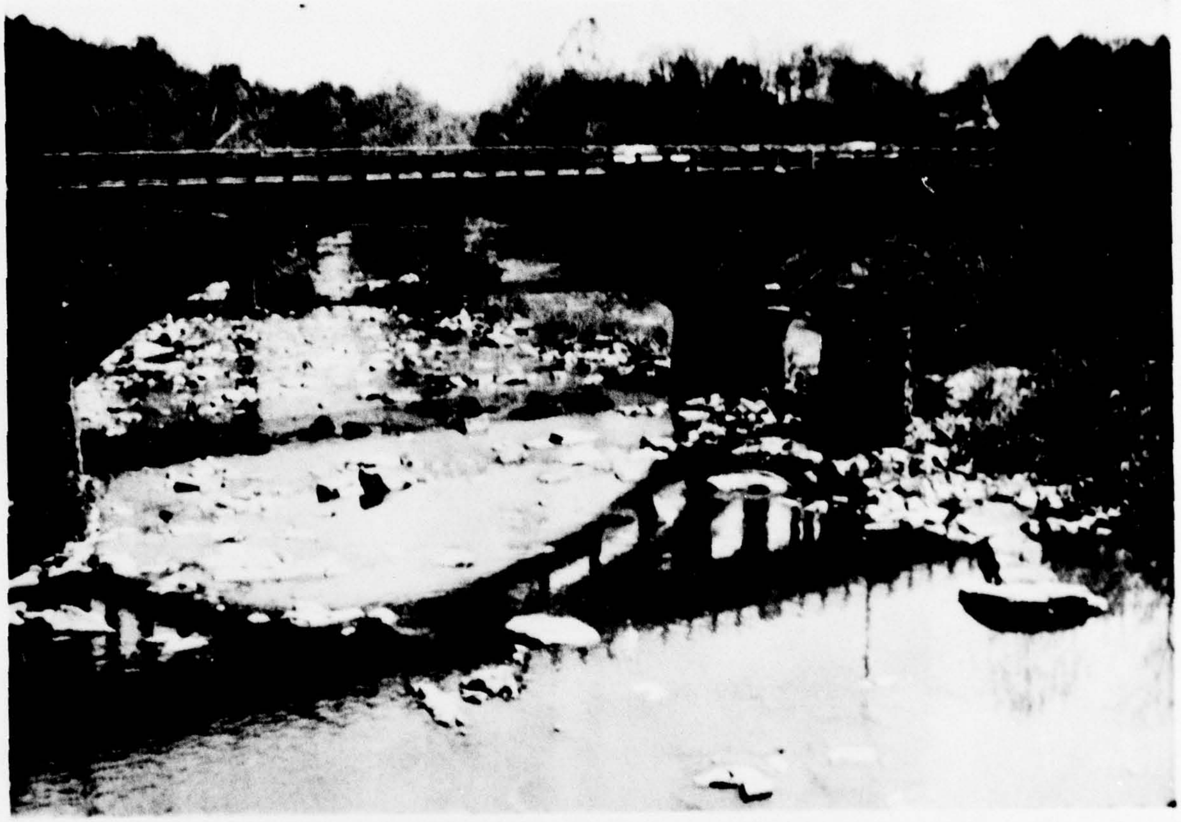
APPENDIX II

PHOTOGRAPHS

CLOSE-UP CATWALK AND GATE SECTION

CLOSE-UP DOWNSTREAM

CLOSE-UP GATE



DOWNSTREAM VIEW

APPENDIX III

FIELD OBSERVATIONS

FIELD OBSERVATIONS

Name of Dam: Smith River Dam, Va. No. 8913

County: Henry

State: Virginia

Coordinates: Lat 36°-39.9' Long 79°-53'

Date of Inspection: May 1, 1979

Weather: Fair, temperature 72°F

Pool Elevation at Time of Inspection: Elevation 694.5 M.S.L.

Tailwater at Time of Inspection: 664.0 M.S.L.

Inspection Personnel:

Schnabel Engineering Associates, P.C.
Ray E. Martin, P.E.
Stephen G. Werner (recorder)

J. K. Timmons and Associates, Inc.
Robert G. Roop, P.E.
William A. Johns (recorder)

City of Martinsville, Virginia
Robert I. Corekin

State Water Control Board
Hugh Gildea

1 Concrete/Masonry:

1.1 Seepage or Leakage: Seepage estimated at less than 1 gallon per minute (gpm) per seep was observed at scattered locations along the left downstream abutment slope. Heavy iron staining, an indicator of long term seepage, was identified with several of these seeps. Minor seepage (less than 1 gpm) was also present along the left auxiliary spillway at Pier No. 4. The water appeared to originate from behind the abutment-slope contact and pass

through deteriorating mortar. The gate between Piers No. 4 and 5 of the auxiliary spillway was boarded up with timbers. Although no seeping water was observed, the accumulation of sediment along and at the base of the timbers indicates seepage has occurred here in the past.

Seepage estimated as up to several gpm was passing through the mortar joints below the auxiliary spillway crest gate betweeen Piers No. 2 and 3 during the inspection. The former gates between the right auxiliary spillway abutment wall and Pier No. 1 and between Piers No. 1 and 2 were boarded up with timbers; however scattered seepage estimated as up to several gpm were observed passing through some of the timber contacts.

Leakage was passing below the spillway crest gates, particularly between Gates No. 7 through 14 numbered from the right abutment. The estimated leakage flow ranged from less than 1 gpm to more than 100 gpm. The greatest flow appeared to occur at Gates No. 8 and 11.

Seepage was also observed on the right abutment behind the powerhouse wall.

1.2 Structure to Abutment/Foundation Junction: The right abutment ties into a steep, natural, heavily wooded slope consisting of slightly to moderately weathered biotite gneiss between the river and access road and highly weathered biotite gneiss and residual soils along and above the access road. The residual soil generally ranges from a

silty sand (SM) to clayey silt (ML). Only minor sloughing was observed along the road cut. The left abutment consists of slight to moderate heavily vegetated slopes which have developed in alluvial deposits. The downstream slope consists of steep brush covered slopes, which exhibit scattered washes of erosional channels. The slope appears to consist of residual soils with scattered weathered bedrock outcrops and fill soils ranging from sand, silt to clay materials which include some rock debris.

1.3 Drains: The lower sluice gate is inoperable.

1.4 Water Passages: Water inlets to the powerhouse are in good condition. Headgates are operable and debris is removed daily.

1.5 Foundation: Bedrock is exposed throughout the downstream channel from the spillway base to the powerhouse. Rock examined in the stream channel consists of slightly weathered, light gray to brown granite with bands of amphibolite or hornblende gneiss and quartz veins. The bedrock contains well developed rectangular joint sets (N25E, 90 and N55W, 90), which are accompanied by periodic obliquely intersection joints. Slightly weathered light gray biotite gneiss occurs along the right abutment near the powerhouse. The depth of tailwater prevented the visual inspection of all outcrops. However, at the high points of dam-foundation contact it appeared the dam is embedded. The depth of this embedment, if any, could not be verified. The contact

between the granite and biotite gneiss occurs along or beneath the dam; however this contact was not located during the inspection.

1.6 Surface Cracks: Grouted rock surface is in good condition except on face of auxiliary spillway where grout is eroded about 3-4 inches from rock surface. A minor rock failure exists between Piers No. 2 and 3 where mortar has eroded away from the rock. Mortar erosion was also observed between Piers No. 3 and 5 in the auxiliary spillway. Overall, surface erosion was observed, but the mortar generally appeared to be good.

1.7 Structural Cracking: No cracking exists on the spillway. The powerhouse has some cracking and spalling of concrete at the rear foundation wall.

1.8 Vertical and Horizontal Alignments: Good.

1.9 Monolith Joints: None.

1.10 Construction Joints: Unseen, surface covered with Gunite.

1.11 Erosion of Abutment Slopes: Occasional minor sloughing was observed along the access road to the powerhouse along the right abutment. Scattered washes or small erosion channels exist on the downstream side of the left abutment adjacent to the Route 220 highway embankment.

2 Gated Spillway:

2.1 Concrete Sill: Fair condition; weathered but structurally sound. Needs grouting.

2.2 Approach Channel: None.

2.3 Discharge Channel: None.

2.4 Bridge and Piers: Good condition. Need painting.

2.5 Gates and Operation Equipment: One gate not operable.

3 Reservoir: In good condition, slight to steep slopes, no debris noticed.

3.1 Slopes: Moderate to steep natural slopes with scattered bedrock outcrops bound most of the right or west side of the reservoir. These slopes are thickly wooded. The left or east side of the reservoir consists of alluvial or stream deposited materials in which shallow slopes have developed. This side of the reservoir is open to thickly vegetated.

3.2 Sedimentation: Sediment build-up is apparent on the left side of the reservoir; however, the balance of the reservoir maintains approximately 23 ft of water.

4 Downstream Channel:

4.1 Condition: Scour appears minimal in the bedrock exposed. The downstream area is characterized by a broad river channel of constant width. Several bridges exist.

4.2 Slopes: Moderately to steeply sloping natural slopes occur on both sides. The slopes are generally vegetated and include scattered wooded areas.

4.3 Population and Facilities: Commercial and industrial development, along with several homes, exist downstream.

5 Instrumentation:

5.1 Monumentation: None.

5.2 Observation Wells and Piezometers: No observation wells or piezometers were located during our field observations.

APPENDIX IV

STABILITY ANALYSIS

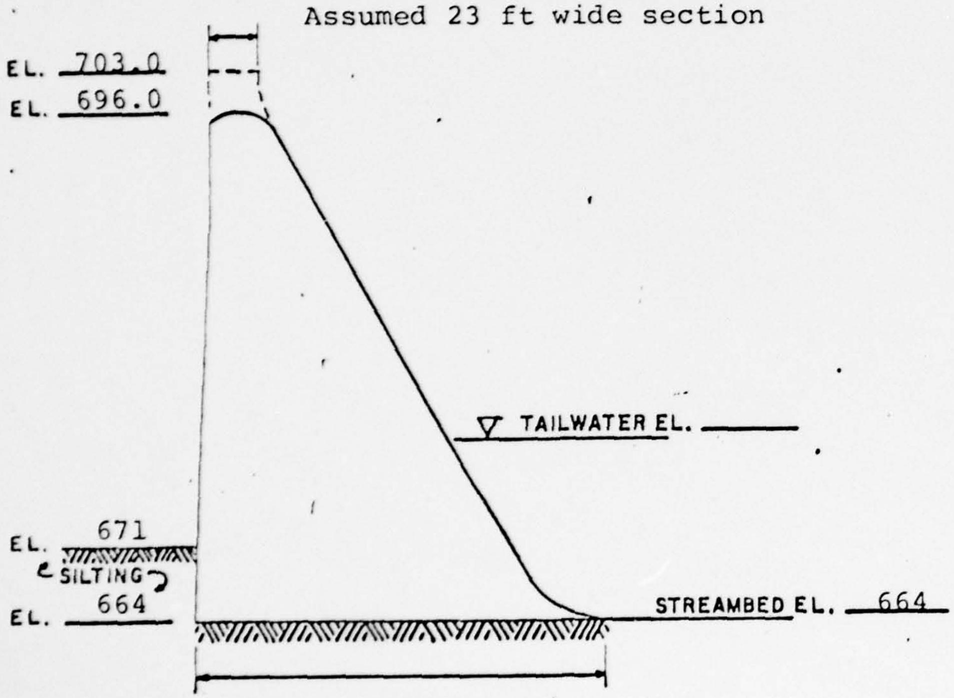
This analysis was performed in accordance with Section 4.4 of Reference 1, Appendix V.

GRAVITY DAM DESIGN
STABILITY ANALYSIS

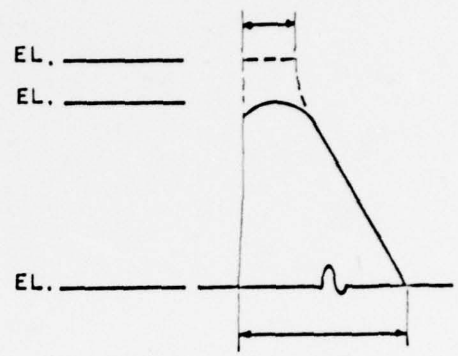
ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION Centerline of Spillway
ANALYSIS PREPARED BY ______

LOADING CASE	ELEV. HEAD WATER	ELEV. TAIL WATER	ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	LOCATION RESULTANT FROM TOE	% BASE IN COMPRESSION	FACTOR SAFETY SLIDING	FOUNDATION PRESSURE TOE	FOUNDATION PRESSURE HEEL
Normal Pool	696	664	767 kips	864 kips	1.13	2.6'	29	1.34	7.78 ksf	—
Probable Maximum Flood	718	716	788 kips	158 kips	0.20	13.3'	100	7.42	1.32 ksf	1.21ksf

Assumed 23 ft wide section



FULL SECTION



PARTIAL SECTION

APPENDIX V - REFERENCES

1. Recommended Guildelines for Safety Inspection of Dams, Department of the Army, Office of the Chief of Engineers, 46 pp.

2. Design of Small Dams, U.S. Department of Interior, Bureau of Peclamation, 1974, 816 pp.

3. Geology of the Martinsville West Quadrangle, Virginia, Report of Investigations 16, James F. Conley and E. Clayton Toewe, Virginia Division of Mineral Resources, 1968, 44 pp.

4. Section 4, Hydrology, Part 1, Watershed Planning, SCS National Engineering Handbook, Soil Conservation Service, U.S. Department of Agriculture, 1964.

5. Hydrometerological Report No. 33, U.S. Department of Commerce, Weather Bureau, U.S. Department of Army, Corps of Engineers, Washington D.C., April 1956.

6. Technical Paper No. 40, U.S. Department of Commerce, Weather Bureau, Washington D.C., May 1961.